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Actual and Estimated Flooding on a Protected Watershed:

Sandstone Creek, Okla.

By
H. N. Holtan
Hydraulic Engineer
Soil Conservation Service

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Rainstorms on Sandstone Creek watershed in Oklahoma after a flood-reduction program had been installed provided an opportunity to check the hydrological procedures used in estimating runoff. They also served as on-the-ground tests of the effectiveness of the flood-control program itself.

Rainfall and runoff during two severe storms in the spring of 1954 have been studied to provide information on the questions:

(1) How nearly correct were the original estimates of the amount of runoff and extent of flooding to be expected from storms of this size after protective measures were installed?

(2) How much did the protective measures reduce flooding from what would have occurred in these same storms without the program?

The data were also used to compare methods used by hydrologists to estimate and route runoff in watershed planning.

This report presents an analysis of the records of the storms of April 29-30 and of May 23-24 which seeks answers to these questions.

In spite of the very uneven distribution of rainfall during these storms the areas actually inundated approximated the estimates for such size storms made during the planning of the program:

Storm	Acres inundated with program	
	Planning estimate	Actual
April 29-30	410	588
May 23-24	310	200

Flows and inundations which would have occurred during these two storms without the program were estimated by two methods. Results of the methods were compared to planning estimates; there was no indication that one method was more accurate than the other.

Storm	Acres inundated without program		
	Planning estimate	Re-estimate by method No. 1	Re-estimate by method No. 2
April 29-30	2,160	2,461	2,335
May 23-24	2,110	2,370	2,259

A comparison of the areas actually flooded by the two storms with the flood-reduction program in effect and the averages of the three estimates of the areas which would have been flooded without treatment indicates the effectiveness of the program.

Storm	Acres inundated with and without program	
	Actual, with program	Estimated, without program
April 29-30	588	2,319
May 23-24	200	2,246

This report describes methods used in analyzing streamflow data. It presents analyses of some of the first records of flows with a flood-reduction program in actual operation during two major storms.

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By
H. N. Holtan, Hydraulic Engineer
Soil Conservation Service
Beltsville, Md.

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Sandstone Creek Watershed

Sandstone Creek rises about 8 miles south of Cheyenne, Okla., and flows in a northeasterly direction, entering the Washita River about 6 miles west of Hammon. It drains about 100 square miles of range and farm land. About 4,700 acres of bottom land (fig. 1) is very important to the agricultural economy of the area.

Before treatment of the watershed, this flood plain was subject to damaging floods an average of 9 times per year. During the 20 years, 1920-39, flooding occurred 184 times. Most disastrous was the Hammon flood of April 1934 when the entire flood plain was inundated. At that time the watershed received 11 inches of rainfall, 8 inches in 2 days.

The Flood-Reduction Program

Supervisors of the Upper Washita Soil Conservation District cooperated with the Washita Valley Flood Control Association and landowners in the watershed to plan and execute a flood-reduction program under the Flood Control Act of 1944. The Soil Conservation Service assisted by making surveys, by recommending soil and water conservation treatments on the land, and by designing and supervising the installation of structures.

The flood-reduction plan included complete soil conservation treatment of the land and the installation of structures planned to control the runoff from 71 percent of the watershed and to protect 95 percent of the flood plain.

Land use and treatment measures were planned in accordance with the Department of Agriculture's objective: "The use of each acre of agricultural land within its capabilities and the treatment of each acre of agricultural land in accordance with its needs for protection and improvement." The planned treatments included seeding 5,800 acres of idle or retired cropland to native grasses; using improved rotations including cover and soil-improving crops on 6,200 acres of cultivated land; building 433 miles of terraces, 50 acres of vegetated waterways, and 17 miles of diversion terraces to help control runoff and erosion on cropland; constructing 125 farm ponds to promote proper distribution of grazing on pasture and range land; building 49 miles of fence; and practicing improved range and pasture management on 58,800 acres including the seeded areas. About 70 percent of these measures had been applied prior to contracting for structural measures in 1950.

The planned structural measures included 24 floodwater-retarding reservoirs (fig. 1), 12 sediment-control dams, and a mile of channel improvement (fig. 2 or 3). These had all been completed, with the exception of some of the channel improvements, by November of 1952.

Estimates of Future Floods

During the preparation of the plan, flood-reduction benefits were estimated. In studying the hydrology of the watershed, the Soil Conservation Service used records of past rainfall to estimate the probability of future occurrence of storms of various magnitudes. Runoff records were analyzed to determine rainfall-runoff relationships for the Sandstone Creek watershed as it was before installation of the program. Historical floodmarks associated with specific storms were used to relate past rainstorms and subsequent runoffs to the acres of land inundated. From these relationships, it was possible to estimate the acres of flood plain which would be inundated by a future storm of a given magnitude if no program were in effect.

To estimate benefits of the program, it was necessary to determine the probable relationship between rainfall and the area which would be inundated with the 24 reservoirs and the land-

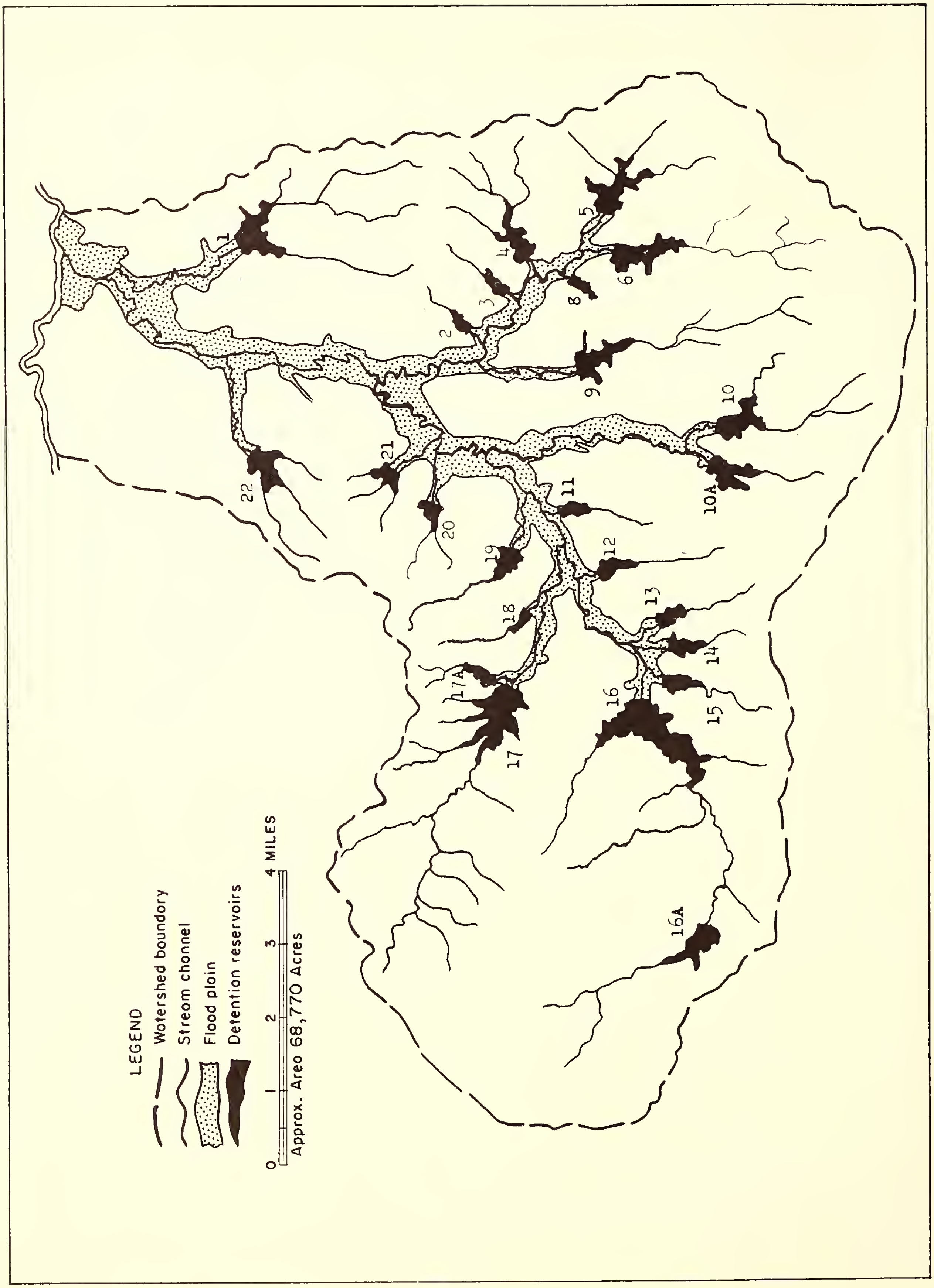


Figure 1.--Flood plain and detention reservoirs on Sandstone Creek.

treatment measures in effect. Storms of various magnitudes were selected for this purpose. Computed runoff from each storm was routed through the reservoirs and channels to calculate high-water elevations. The areas of flood plain below these elevations were related to the total runoff and subsequently to rainfall amounts, to provide a basis for estimating the areas which would be inundated by storms of given magnitudes with the reservoirs and other protective measures installed. Future benefits were then estimated as the differences in areas inundated with and without the program for storms of various magnitudes multiplied by the probable frequency of such events.

The accuracy of these estimates could be tested only by the occurrence of flood-producing storms with the program in effect. The storms of April 29-30 and May 23-24, 1954, provided the opportunity to study the functioning of the protective system and to compare actual flows and inundations with the estimates made in planning.

Instrumentation of Watershed

To provide a means of checking these work-plan estimates, the watershed was equipped with instruments for the measurement of rainfall and for runoff determination.

Stage recorders were installed and operated by the U. S. Geological Survey under cooperative agreement with the Soil Conservation Service at 3 strategic points in the channel system and at 11 of the 24 reservoirs to provide continuous records of water stages during each storm (figs. 2 and 3). The gaged reservoirs were selected to provide good geographic representation of the watershed and to represent the diversity of size, land use, and soils of the separate drainage areas. The records provide an excellent basis for deriving synthetic hydrographs for the 13 ungaged reservoirs. All the gages operated perfectly during the storm of May 23-24, but some failed during the storm of April 29. Only 6 of the 11 reservoir gages provided inflow records for the April storm which could be analyzed.

Fourteen recording and twenty-five nonrecording rain gages were installed in cooperation with the U. S. Weather Bureau. They were located to provide continuous records of rainfall on various parts of the watershed (figs. 2 and 3). The rainfall records were used in the analysis of the 3 stream-gage records, the runoff records from the 11 reservoir-stage recorders, and in the synthesis of hydrographs for the 13 ungaged reservoirs.

Crest gages were installed to permit accurate determinations of maximum stages and subsequent inundations along the entire flood plain. Provisions were made for the accurate survey of areas inundated by future streamflows.

Record of Two Storms

The Sandstone Creek watershed received a total of 4 inches of rainfall during April 25, 27, and 29, 1954, following a 2-week period of no rainfall. In the first half of May a succession of rains totaling 3.4 inches occurred and on May 23-24 another 4.36 inches of rain fell. These storms were of intensity and magnitude to produce flooding. Total runoff as computed from stage records was the equivalent of 1.39 inches over the entire watershed from the April 29 storm and 1.35 inches from the May 23-24 storm.

Extent of Flooding With Program

Highwater marks and detailed surveys indicated the areas of flood plain which were actually inundated by each storm. These data afford an opportunity to compare estimates developed during the planning of the program with the records of two actual events:

	Estimated and actual inundation with program in effect				
Storm	Runoff	Estimated[1]		Surveyed[2]	
	Inches	*Acres*	*Percent flood plain*	*Acres*	*Percent flood plain*
April 29	1.39	410	9	588	13
May 23-24	1.35	310	7	200	4

[1] Estimates assumed uniform runoff from entire watershed.
[2] Influenced by unequal distribution of rainfall and runoff.

Several factors account for the discrepancies between estimated and actual flooding. During the April storm, log and debris jams occurred. Also, as can be observed from the isohyetal map (fig. 2) the heaviest rainfall was centered over the uncontrolled area below the reservoirs. Because of these factors the actual inundation was greater than that predicted for a storm of this size.

During the May storm, the opposite occurred. The isohyetal map (fig. 3) indicates heaviest rainfall on the controlled area above the reservoirs. The planning estimate was higher than the record of actual flooding because a greater than normal part of the runoff passed through the reservoirs and emerged at controlled rates.

Discrepancies between estimated and surveyed inundations with the program in effect indicate that the distribution of the rainfall over the watershed influenced runoff. Rainfall distribution might have had an even greater influence on peak flows and inundation if the moderating influence of the program had been absent.

Estimating Flooding Without Program

Relationships between inundation and stage developed during the preparation of the plan were used also to estimate the areas of flood plain which would have been inundated by storms the size of these without the program. It was estimated

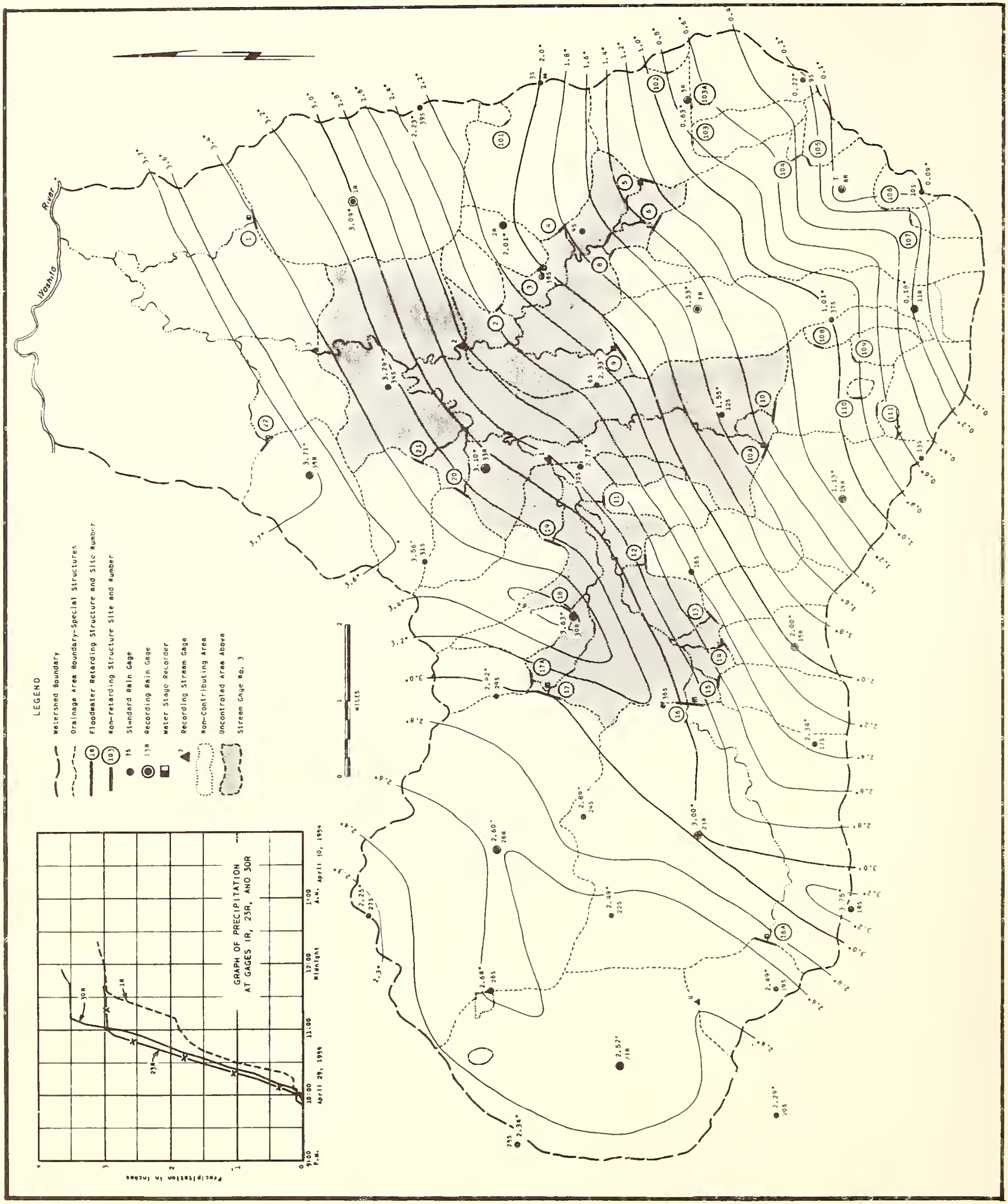


Figure 2.--Isohyetal map, rainfall from April 29, 1954, at 10:00 p. m. to April 30, 1954, at 12:20 a. m., Sandstone Creek watershed of the Washita River watershed, Oklahoma and Texas.

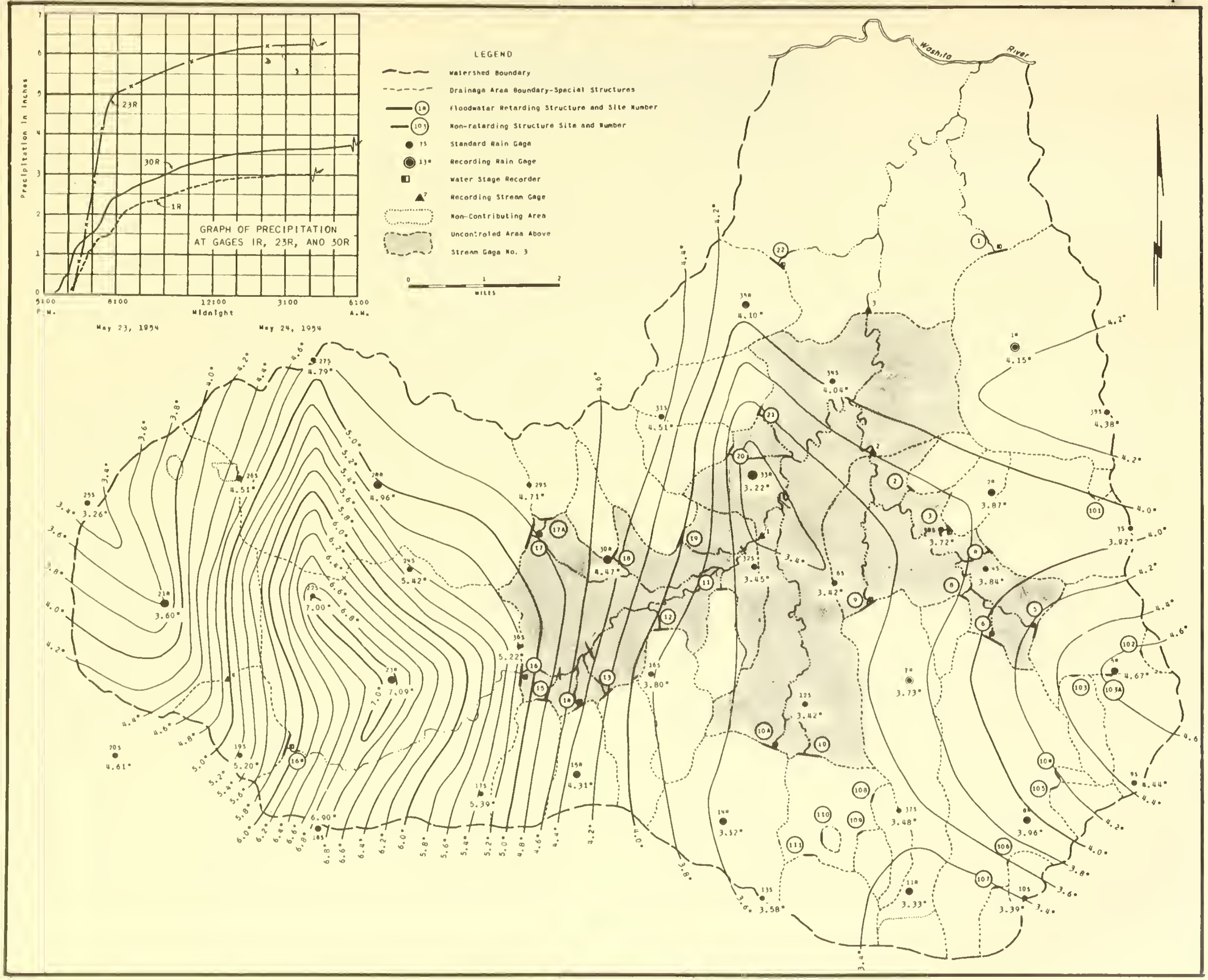


Figure 3.--Isohyetal map, rainfall from May 23, 1954, at 6:00 p. m., to May 24, 1954, at 6:30 p. m., Sandstone Creek watershed of the Washita River watershed, Oklahoma and Texas.

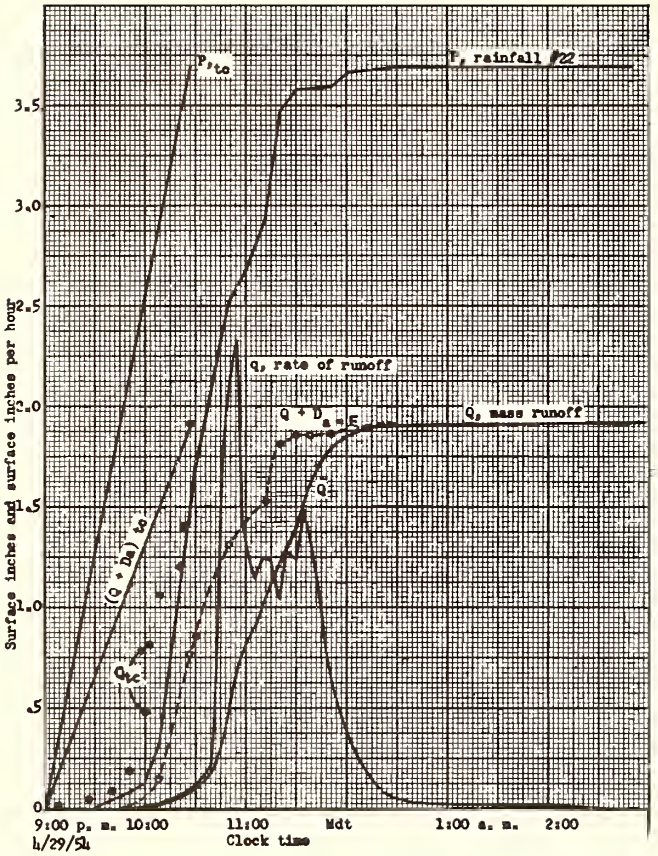
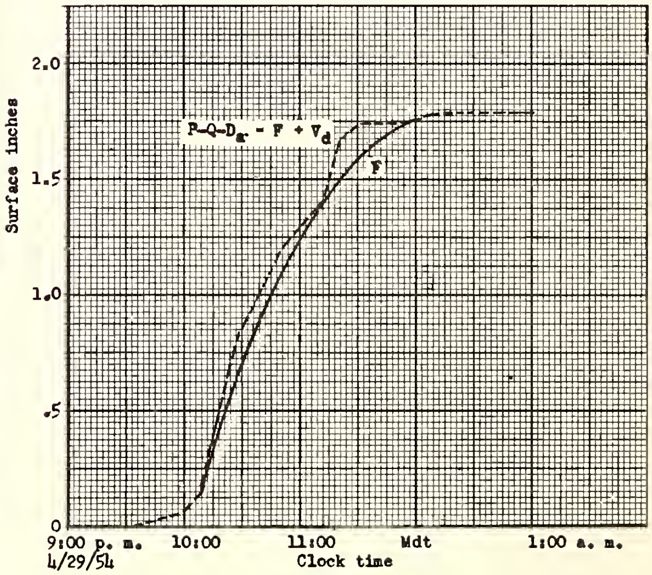


Figure 4.--Analysis of hydrograph of inflow to reservoir No. 22.

Figure 5.--Estimate of mass infiltration from hydrograph of inflow to reservoir No. 22.



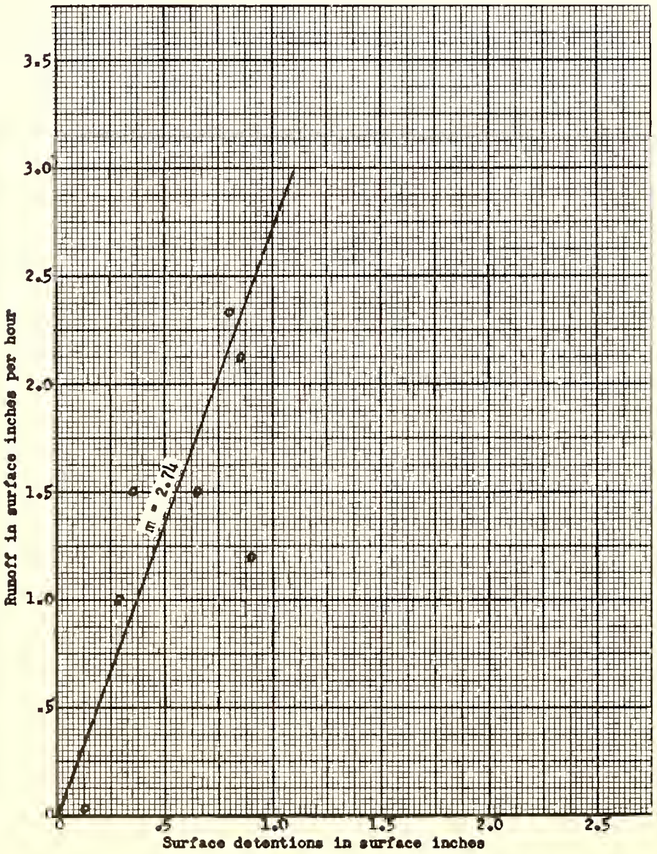


Figure 6.--Storage-indication curve for drainage area above reservoir No. 22.

that about 2,160 acres would have been flooded by a storm the size of the April storm and 2,110 acres by one the size of the May storm.

These planning estimates included allowance for the effects of land treatment, which were assumed to diminish as the size of the storm increased. Records obtained during the April and May 1954 storms were used with no adjustment for land-treatment effects to estimate inundations which would have occurred without the structural program. Two methods were used in routing subarea runoff directly through the channel system without the reservoirs. Land treatments would have little effect upon total runoff for storms of this size on this watershed, for effects of these measures on peak flows diminish as the soil becomes saturated. Although control of erosion and sedimentation continues, the saturated soil cannot continue to absorb appreciable amounts of rainfall unless it can dispose of equal amounts by seepage to underground storage or to the surface at lower elevations.

The changes in land use on the Sandstone Creek watershed were moderate in nature and had not been in effect long enough to have appreciably improved the capacity of the soils to absorb rainfall. Therefore, their effects on runoff during the 1954 storms were considered indeterminate from the data obtained. Flooding without the program was estimated by routing reservoir inflows unadjusted for land treatment through the channel system without routing through the reservoirs.

Records of inflow to 11 of the reservoirs together with detailed measurements of rainfall during the 2 storms provided good bases for determining the amounts and rates of reservoir inflow produced on the other 13 controlled areas. Records of streamflow were used to determine local inflows (runoff from uncontrolled areas). They were also analyzed to provide data for use in the continuity equation by which subarea flows can be routed through the channel system to derive estimates of peak flows and inundations.

Estimating Inflow to Ungaged Reservoirs

The records of inflow to the 11 reservoirs with stage recorders were analyzed to provide data for estimating inflow to the 13 ungaged reservoirs. The analysis of the inflow record at reservoir No. 22 on April 29 (fig. 4) illustrates the type of analysis applied to each runoff record.

Analysis of data from gaged reservoirs

The U. S. Geological Survey converted the reservoir-stage records to inflow hydrographs before transmitting them to the Soil Conservation Service. To produce the inflow hydrographs, increments of pondage were converted to volumes and then to rates of inflow and added to the outflow through the principal spillway. Corrections for rainfall on the pool and for losses to seepage were applied to produce the inflow hydrographs as represented in figure 4 by curves for rate of inflow (q) in inches per hour and accumulated inflow (Q) in inches for the drainage area. Accumulated rainfall in inches is plotted as curve P. These data were analyzed to derive an accumulated infiltration curve (F) for application to ungaged areas. They were further analyzed to derive an equation for routing excess rainfall on each area to produce a hydrograph of runoff.

There are a number of analytical methods which could have been used for this purpose; for simplicity, the "time condensation[1]" method was used to estimate the status of detention during the period of runoff.

The difference between the mass rainfall curve (P) and the runoff-plus-detention curve ($Q + D_a$) is plotted in figure 5 as $F + V_d$. The $F + V_d$ curve represents retention and is composed of infiltration (F) plus volume of depression storage (V_d). Experience has indicated that mass infiltration curves are smooth curves of diminishing slope. The F curve therefore is shown as a smoothed curve, the rest of the volume under the $F + V_d$ curve being attributed to depression storage. Since rainfall was excessive during this period, the F curve of figure 5 represents infiltration capacity.

The detention storage indicated in figure 4 as the difference between curve $Q + D_a$ and curve Q is also used to provide a storage equation for use in routing excess rainfall to synthesize hydrographs of runoff. Figure 6 is a plotting from figure 4 of rate of runoff against average detention storage, read at various clock times. It is evident that the relationship of rate of runoff to detention storage is actually a loop curve with flows per unit of detention less on the rising side of the hydrograph than on the receding. However, for practical purposes the relationship is expressed as one straight line. The slope of this line $m = 2.74$ means that on this subarea 2.74 inches per hour of runoff is produced for each inch of detention storage. The equation[2] is $q = m\ D_a$.

Similar analyses were made of the records from reservoired areas No. 9, 14, 16, 16A, and 17. Infiltration-capacity curves are presented in figure 7 and detention-flow relationships in figure 8. Each ungaged reservoir-drainage area for which a hydrograph was to be synthesized was compared to these six areas to select the one most similar in regard to infiltration. A geologic-soils map of the watershed was prepared (fig. 9) to aid in these comparisons. When a representative infiltration curve had been chosen, a second comparison was made to choose a representative detention-flow relationship.

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[1] Holtan, H. N. Time condensation in hydrograph analysis. Amer. Geophys. Union Trans. 26: 407-13. 1945.

[2] All symbols used in equations and graphs are shown in one alphabetical list on the last page.

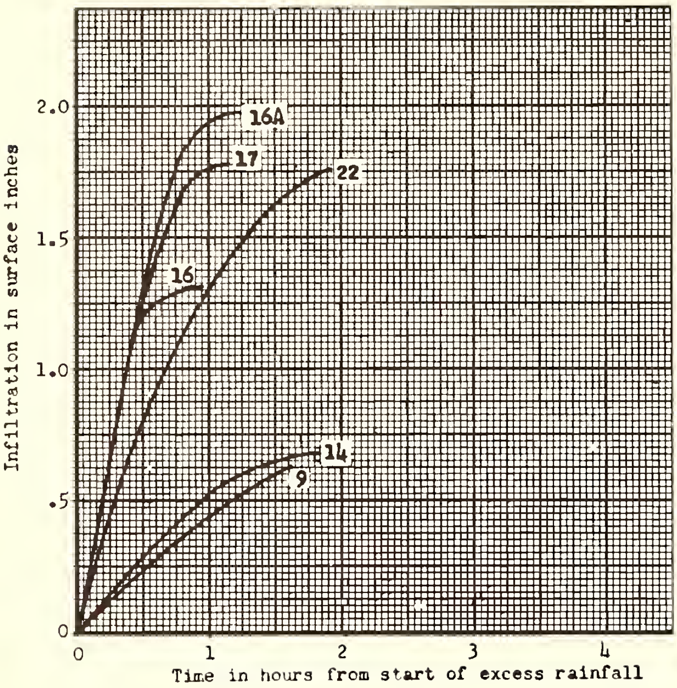
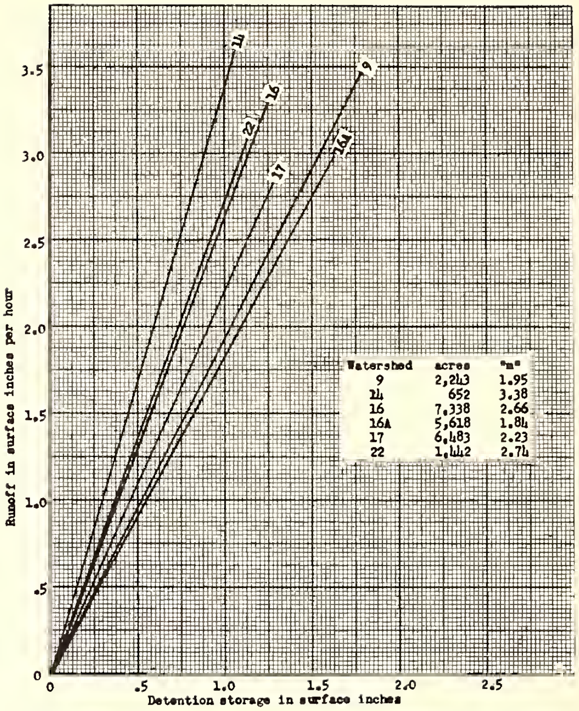


Figure 7.--Curves of mass infiltration derived from analyses of inflow hydrographs.



Watershed	acres	"m"
9	2,243	1.95
14	652	3.38
16	7,338	2.66
16A	5,618	1.84
17	6,483	2.23
22	1,442	2.74

Figure 8.--Detention storage-indication curves for drainage areas above six reservoirs (from hydrograph analyses).

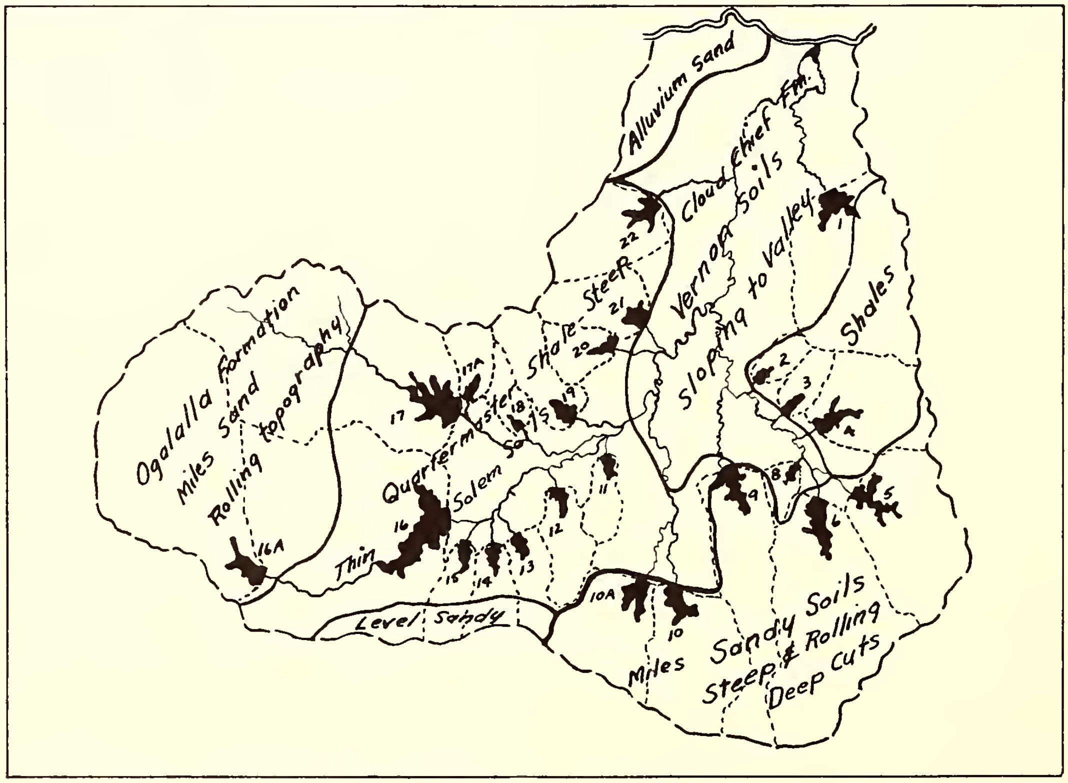


Figure 9.--Geology, soils, and topography on Sandstone Creek.

Synthesis of hydrographs for ungaged reservoirs

Hydrographs were synthesized for ungaged areas by the procedures illustrated by figure 10 for area No. 18. First, rainfall for the area was plotted. Then an infiltration curve was selected from those derived from the hydrographs of the gaged areas (fig. 7) to be used to represent area No. 18. The only gaged area lying entirely within the same geologic and soils area as No. 18 is No. 22 (fig. 9). Therefore, infiltration curve No. 22 was superimposed upon the rainfall curve of area No. 18 (fig. 10), with baselines coincident. The F curve was shifted so that no excess rainfall was indicated until the slope of the rainfall curve exceeded that of the infiltration curve. Thereafter, infiltration was assumed to continue at capacity rate during all periods when rainfall rate exceeded infiltration capacity. During periods of rainfall at rates less than infiltration capacity, actual infiltration was limited to rainfall plus estimated depression storage. In figure 10, depression storage was estimated at 0.30 inch. The F curve was extended beyond the essential cessation of rainfall (at 11:10 p. m.) by this amount plus the 0.11 inch of subsequent rainfall.

The difference between rainfall and infiltration at any instant is the total excess rainfall up to that time. Theoretically, this is the amount of water available for runoff. But a small part of this water is in depressions and would never occur as runoff. Also, part of that occurring as surface detention infiltrates as runoff continues. The depletion of the supply of water available for runoff is shown by the recession of the P-F curve. The runoff is calculated by apportioning between detention storage and runoff the supply of excess rainfall (P-F).

Computations in the routing of runoff from area No. 18 are shown in table 1. Excess rainfall (E) was tabulated by 15-minute intervals in column 2. Increments of excess rainfall were converted to average inflow (I_a) in column 3 by the equation:

$$I_a = \frac{\Delta E}{\Delta t} A$$

Excess rainfall was then routed to produce the runoff hydrograph. Experience indicates that area and topography of the watershed are two of the most significant factors affecting the relation of detention storage to rate of flow. Of the six watersheds for which storage-indication curves are shown in figure 8, No. 14 (652 acres) is nearest in size to No. 18 (390 acres). The geology and soils map (fig. 9) indicates that area No. 14, unlike No. 18, includes a considerable portion of level land. Nevertheless, No. 14 appeared to be the best choice among the available detention storage-indication curves.

Table 1.--Routing excess rainfall by the watershed detention storage-indication method to produce a hydrograph for drainage area above reservoir No. 18, April 29-30, 1954

1	2	3	4	5
Time	E	I_a	$\frac{S}{\Delta t} + \frac{O}{2}$	O
	Inches	*Average c. f. s.*	*C. f. s.*	*C. f. s.*
10 p. m.	0	0	0	0
10:15	.11	172	172	102
10:30	.48	577	647	384
10:45	.85	577	840	499
11:00	1.30	702	1,043	620
11:15	1.80	780	1,203	713
11:30	1.67	-203	287	170
11:45	1.62	-78	39	22
12:00	1.60	-31	0	0
12:15 a. m.	1.55	-78		
12:30				

For routing purposes the continuity equation which states that inflow equals outflow plus change in storage may be written as follows:

$$I_a \Delta t = \left(\frac{O_2 + O_1}{2}\right) \Delta t + (S_2 - S_1) \qquad (1)$$

wherein subscripts 1 and 2 refer to beginning and ending of Δt.

A more convenient form for tabular solution, as in table 1, is:

$$I_a + \left(\frac{S_1}{\Delta t} + \frac{O_1}{2}\right) - O_1 = \left(\frac{S_2}{\Delta t} + \frac{O_2}{2}\right) \qquad (2)$$

For any particular increment of time, Δt, the expression $\frac{S_2}{\Delta t} + \frac{O_2}{2}$ is the only unknown and can therefore be obtained by solution as was done in column 4 of table 1.

Values of outflow (O) in column 5 are read from a plotting of (O) against $\left(\frac{S}{\Delta t} + \frac{O}{2}\right)$

In order to use equation (2) as it is written the detention-storage-flow relationships of figure 8 were converted from the form $q = m D_a$ to a plotting of O against $\left(\frac{S}{\Delta t} + \frac{O}{2}\right)$ by the following steps:

1. Since 1 inch per hour per acre = 1 cubic foot per second, ... $O = qA$.
2. Since $\frac{D_a A \Delta t}{\Delta t}$ = storage in cubic feet per second x hours, ... $S = D_a A$.
3. Therefore, ... $\left(\frac{S}{\Delta t} + \frac{O}{2}\right) = \left(\frac{D_a A}{\Delta t} + \frac{qA}{2}\right)$.
4. Since $q = m D_a$, $\left(\frac{S}{\Delta t} + \frac{O}{2}\right) = A\left(\frac{D_a}{\Delta t} + \frac{m D_a}{2}\right)$.

5. By transposing .... $D_aA = \dfrac{\left(\frac{S}{\Delta t} + \frac{O}{2}\right)}{\left(\frac{1}{\Delta t} + \frac{m}{2}\right)}$.

6. Then, since $O = qA = m\ D_aA$,

$$O = \left(\frac{m}{\frac{1}{\Delta t} + \frac{m}{2}}\right)\left(\frac{S}{\Delta t} + \frac{O}{2}\right) \qquad (3)$$

This method of routing excess rainfall through the entire drainage system in one operation is hereafter referred to as the watershed detention storage-indication method. Equations (2) and (3) are alternately applied to derive the quantities $\left(\frac{S}{\Delta t} + \frac{O}{2}\right)$ and outflow (O) respectively.

For area No. 14, m = 3.38; using $\Delta t = 1/4$, equation (3) becomes:

$$O = \frac{3.38}{4 + \frac{3.38}{2}}\left(4S + \frac{O}{2}\right),$$

Or

$$O = 0.594\left(4S + \frac{O}{2}\right).$$

This relationship was plotted in figure 11 and subsequently used to compute columns 4 and 5 of table 1. The resulting hydrograph is plotted in figure 10.

Hydrographs were developed similarly for the other ungaged reservoirs. These are superimposed on a schematic diagram of the drainage system showing points of confluence of the tributaries (figs. 12 and 13).

Estimating Local Inflow

Local inflow is used here to mean runoff from those areas which were not controlled by reservoirs. Estimates of local inflows were derived from the streamflow records obtained at the three gaging stations on Sandstone Creek.

The streamflows observed at each of the three gaging stations consisted of both local runoff and reservoir outflows. Since reservoir outflow was regulated in each case by the size of the outlet tube, it was readily recognized in each streamflow hydrograph as the sustained near-constant flow during the recession of the curve. The hydrograph of local runoff was obtained by subtracting reservoir outflows from the observed streamflow at each station. Reservoir outflow was assumed to be constant from the time reservoir inflow started until the end of the streamflow hydrograph.

Flows past gages No. 1 and No. 2 are also included in those passing gage No. 3. Local inflows from areas lying below the first two stations but above No. 3 were obtained by subtraction of hydrographs.

Each of the hydrographs of local runoff thus obtained was analyzed to determine rainfall excess (E) as was done with the reservoir inflow hydrographs (fig. 4). These curves of rainfall excess were used as inflow hydrographs for routing flows through the drainage system.

Routing Runoff to Compute Streamflows

Inflows were routed downstream to estimate streamflows which would have occurred during the two storms if the flood-reduction program had not been in effect. For comparison, two methods of flood routing were used:

(1) The reach storage-indication method of routing through successive channel reaches by equation (2), using physical measurements of the valleys and stream to derive storage-indication curves for each reach. This is a modification of the Puls method[3] developed by the U. S. Corps of Engineers and widely used in flood routing.

(2) The watershed detention storage-indication method, using recorded volumes and rates of runoff to compute one storage-indication curve for the entire drainage area above the point of observation. The detention storage-indication curve includes detentions of overland flow as well as of streamflow. This method was used to compute hydrographs by equations (2) and (3) at stream-gage locations for comparison with those obtained by the reach method.

Reach storage-indication method

The reach storage-indication method involves calculations for successive segments or reaches of the stream system (fig. 12). Since the cross-section data were not related to a common elevation datum, it was not possible to compute water-surface profiles. Consequently, "cross section rating curves" were computed by the slope-area method to show rate of flow in relation to stage or depth in each reach. Storage was then computed for each reach as the product of length and average end-area. The relationship of storage to rate of flow was curvilinear in each case.

Reservoir outflows for these storms could be fairly well determined and associated with specific reservoirs since the sustained recession flows recorded at the stream gages were essentially accounted for by gaged reservoir outflows. Local inflows as determined by analysis of stream-gage hydrographs were apportioned to the reaches according to local inflow drainage areas.

The routing data and procedures were first tested in an application of equation (2) to reproduce the hydrographs which actually occurred. Local inflows and reservoir outflows were systematically combined and routed downstream by equation (2) and by reference to storage-indication

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[3] U. S. Corps of Engineers. Flood Control. Engin. construction text X-156. Engin. School, Ft. Belvoir, Va. p. 134. 1940.

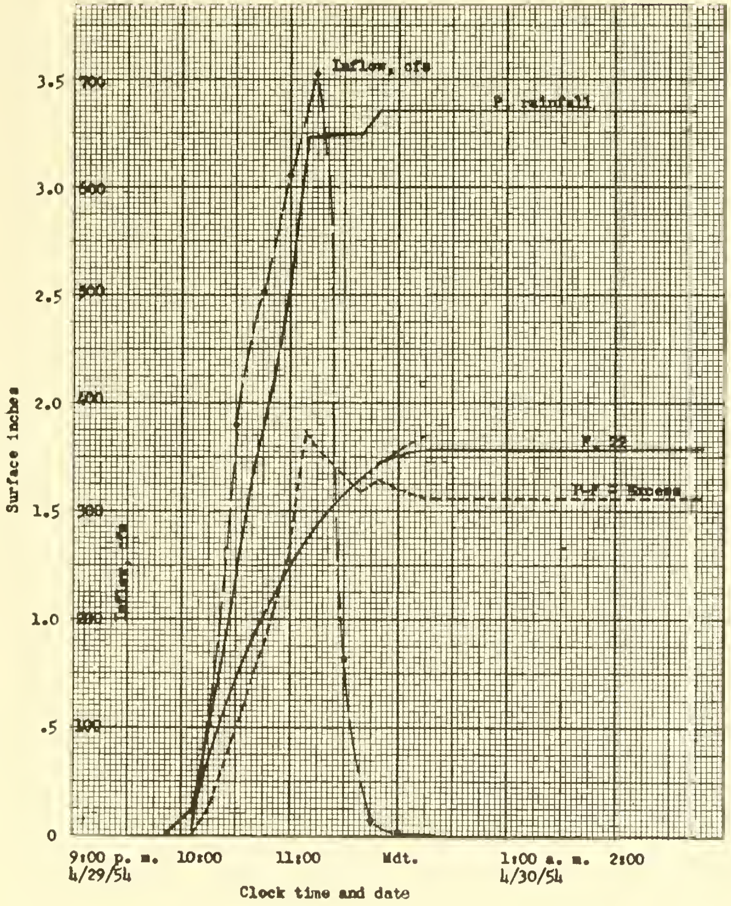

Figure 10.--Derivation of inflow hydrograph for reservoir No. 18.

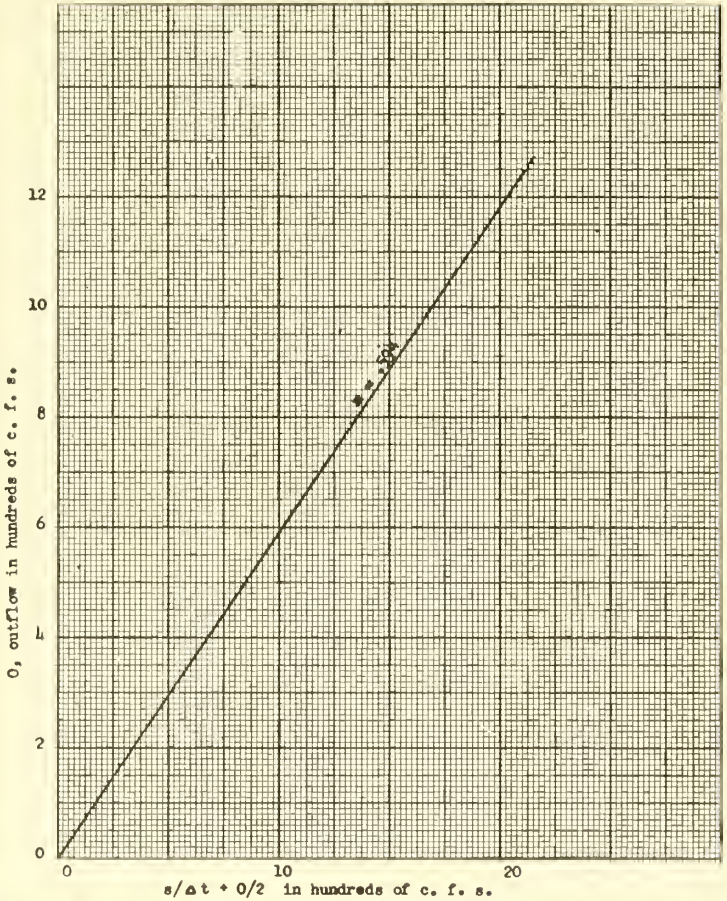

Figure 11.--Detention storage-indication curve for drainage area above reservoir No. 18.

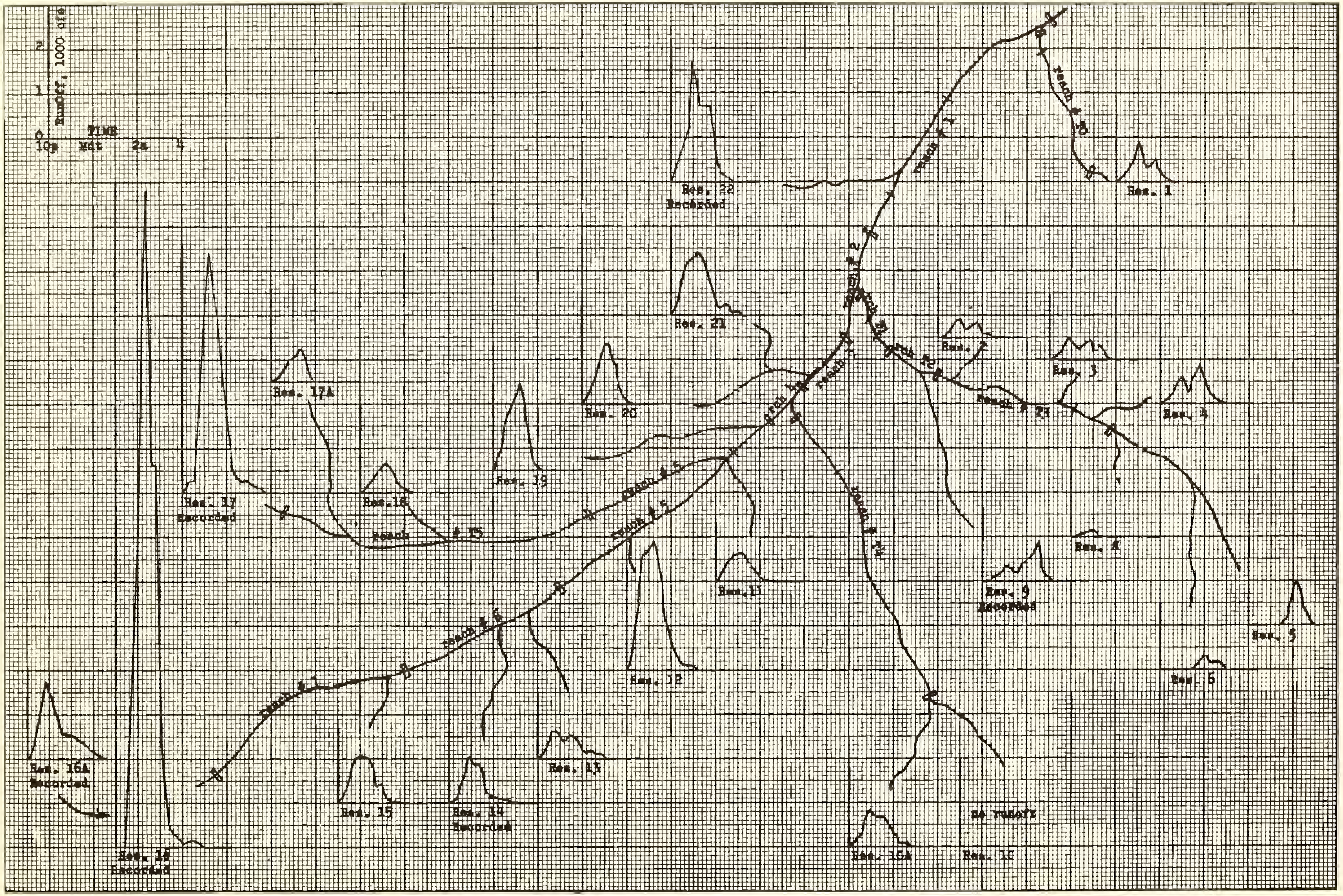

Figure 12.--Hydrographs of inflow to reservoirs during the storm of April 29 and 30, 1954.

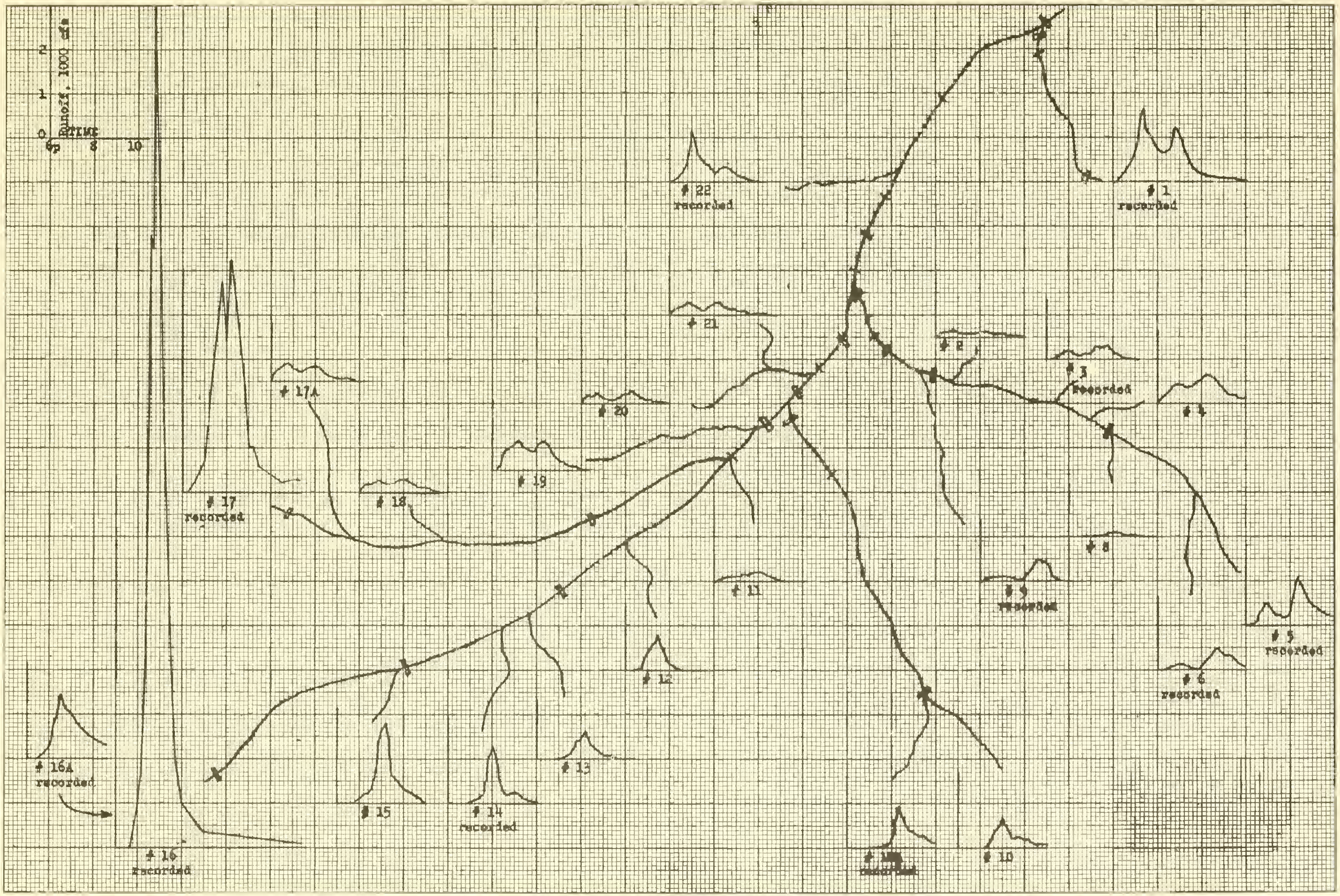

Figure 13.--Hydrographs of inflow to reservoirs during the storm of May 23 and May 24, 1954.

curves (not shown). Hydrographs of streamflow computed at each gage site compared quite well with the hydrographs of observed flows (figs. 14, A and 15, A). The hydrographs for stream gage No. 2 were too small to afford a good comparison, hence are not shown.

Due to lack of data, it was not always possible to route the flow from a reservoir down to the principal stream. If the length of the tributary was significant, the calculations were made as though the point of entry had been at the upper end of the channel reach. Routing over this extra distance in the main stream compensated to some extent for the omission of routing through lengths of tributaries below the dams (figs. 14, A and 15, A). This procedure proved satisfactory.

Using the same system, reservoir inflow hydrographs were substituted for outflow hydrographs, combined, added to local inflow, and routed through the successive stream reaches. Hydrographs thus produced indicate the flows which would have occurred from these two storms had the reservoirs not been installed (figs. 14, C and 15, C).

Watershed detention storage-indication method

The watershed detention storage-indication method considers detention storage over the entire watershed at one time, rather than in successive reaches of the stream channel and valley. The hydrographs recorded at the stream gages were analyzed by the method previously explained (fig. 4) to derive detention storage-indication curves. As often happens when storage includes both overland flow and streamflow, these relationships were linear. Therefore, the slope-intercept formula was applied to derive values of m for substitution in equation (3).

Using the 1/4-hour time increment, the values of the factor $\dfrac{m}{\left(\dfrac{1}{\Delta t} + \dfrac{m}{2}\right)}$ were:

Gage No. 1, 0.312; No. 2, 0.212; No. 3, 0.113.

As a check on these values, equation (3) was first applied to local inflow plus reservoir outflows to develop hydrographs of streamflow at each gaging station for the April and May storms. The results compared satisfactorily with the observed hydrographs (figs. 14, B and 15, B).

Using the same factor values, equation (3) was applied to local inflows plus reservoir inflows to compute the hydrograph at each gaging station for conditions before the reservoirs were installed. In the absence of reservoirs, the inflows would enter the channel system directly; hence the inflow data were used instead of reservoir outflows in this computation. Derived hydrographs are plotted in figures 14, C and 15, C for comparison with those computed by routing through successive reaches.

Comparison of methods

Hydrographs of the floods which would have occurred had the reservoirs not been installed at the time of the April and May storms, as calculated by the two routing methods, are compared in figures 14, C and 15, C. There is no real basis for deciding which is the more accurate. Comparison of travel time from gage No. 1 to No. 3 and from No. 2 to No. 3 is not necessarily significant since the peak flow at No. 3 may or may not have been appreciably influenced by inflows below the other two gages. Both methods indicate a lesser peak flow at gage No. 3 than at No. 1 for both storms. Peak rates determined by the two methods have a maximum of 8 percent departure from the average of the two. Since there is no basis for selection, both methods were considered in estimating the areas inundated.

Areas Inundated

Areas which would have been inundated at the peak of each flood without watershed treatment were estimated by reference to stage-discharge and stage-inundation relationships which had been plotted for each cross section. Peak rates of flow at the foot of each reach were read from the hydrographs developed by the reach storage-indication routings. Peak rates for points within the reach were estimated by apportioning according to drainage area. The corresponding areas of inundation were totaled. These estimates are compared in table 2 with corresponding estimates developed in preparing the work plan. Inundated areas as calculated by the reach storage-indication method are 14 percent higher than the planning estimates for the April storm and 12 percent higher for the May storm.

Table 2. --Estimated inundation without watershed protection

Method used in estimating	Area inundated	
	April storm	May storm
	Acres	*Acres*
1. Method used in work plan[1]	2, 160	2, 110
2. Reach storage-indication [2]..........	2, 461	2, 370
3. Watershed detention storage-indication[2]	2, 335	2, 259
Average	2, 319	2, 246

[1] Assumed equal distribution of rainfall and runoff.

[2] Recognized actual distribution of rainfall and runoff through use of specific storm data.

The watershed detention storage-indication method produced hydrographs only at gage locations. Consequently peak flows at other cross sections had to be estimated. This was done by adjusting the peaks at intermediate points obtained by the reach storage-indication method in proportion to differences between the peaks as computed by the two methods at the stream-gage locations. The peaks thus adjusted were used to compute areas of inundation. These differed from the results of the work-plan method by 8 percent

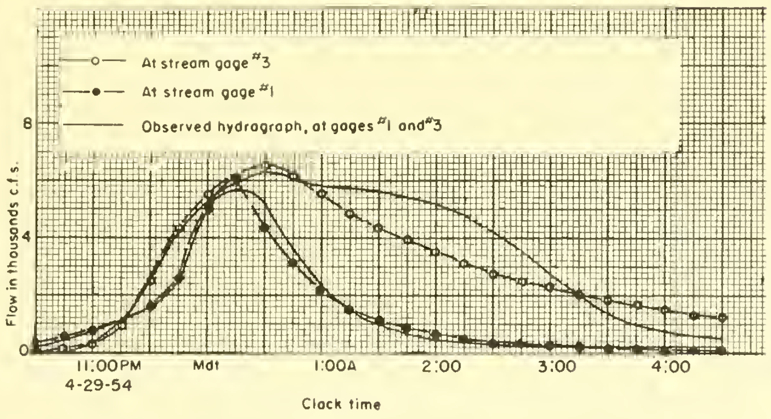

Figure 14 B.--Hydrographs derived by the watershed detention storage-indication method and actual hydrographs for the storm of April 29 and 30, 1954, with reservoirs.

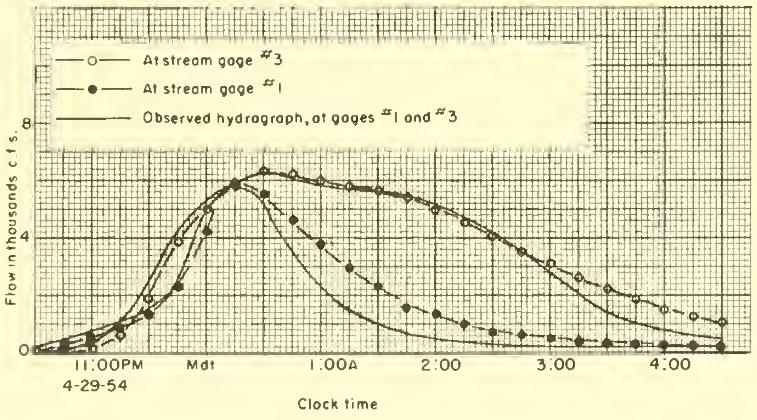

Figure 14 A.--Hydrographs derived by the reach storage-indication method and actual hydrographs for the storm of April 29 and 30, 1954, with reservoirs.

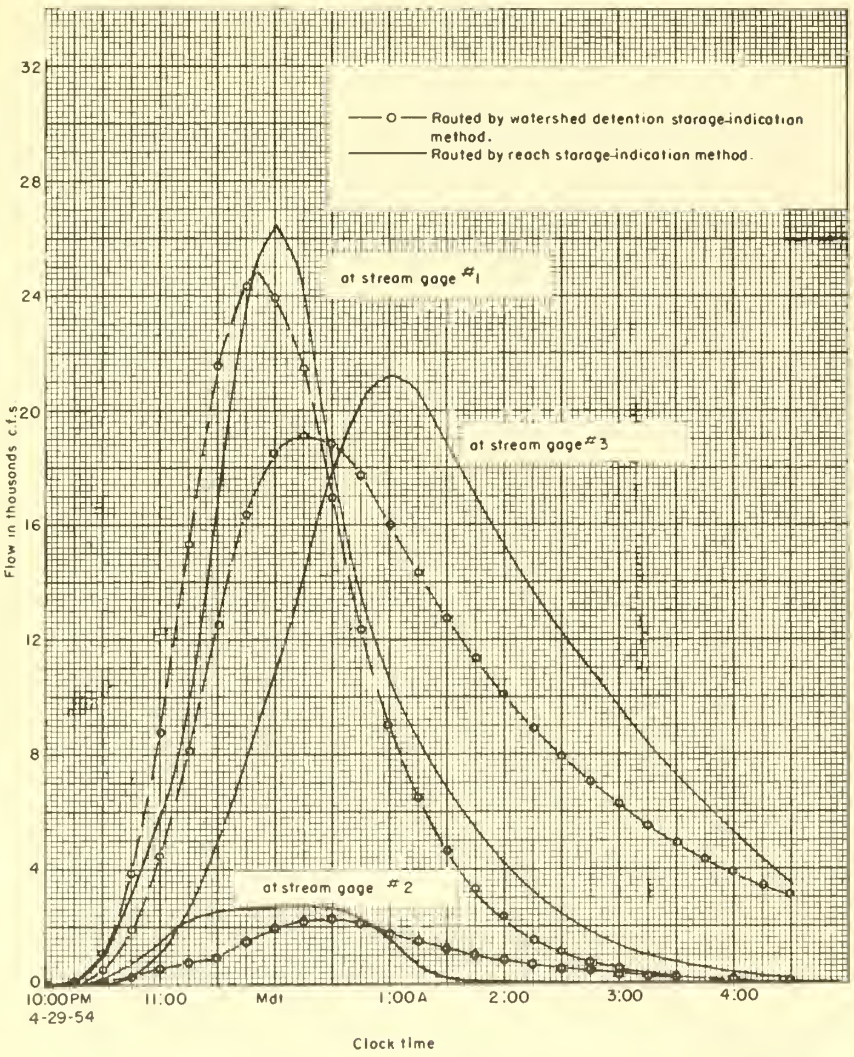

Figure 14 C.--Hydrographs which would have occurred during the storm of April 29 and 30 if the reservoirs had not been installed.

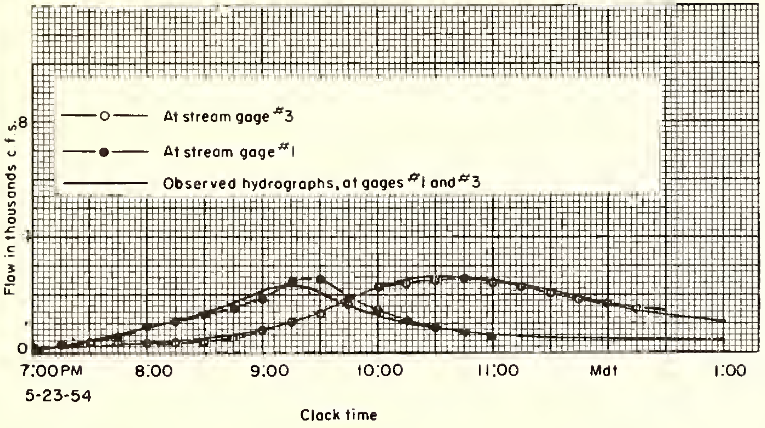

Figure 15 B.--Hydrographs derived by the watershed detention storage-indication method and actual hydrographs for the storm of May 23 and 24, 1954, with reservoirs.

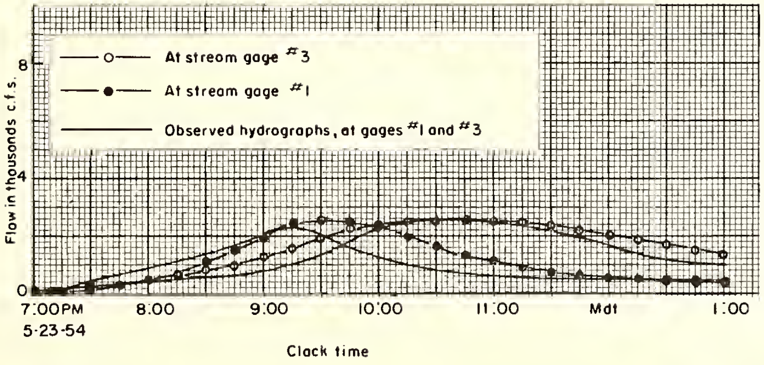

Figure 15 A.--Hydrographs derived by the reach storage-indication method and actual hydrographs for the storm of May 23 and 24, 1954, with reservoirs.

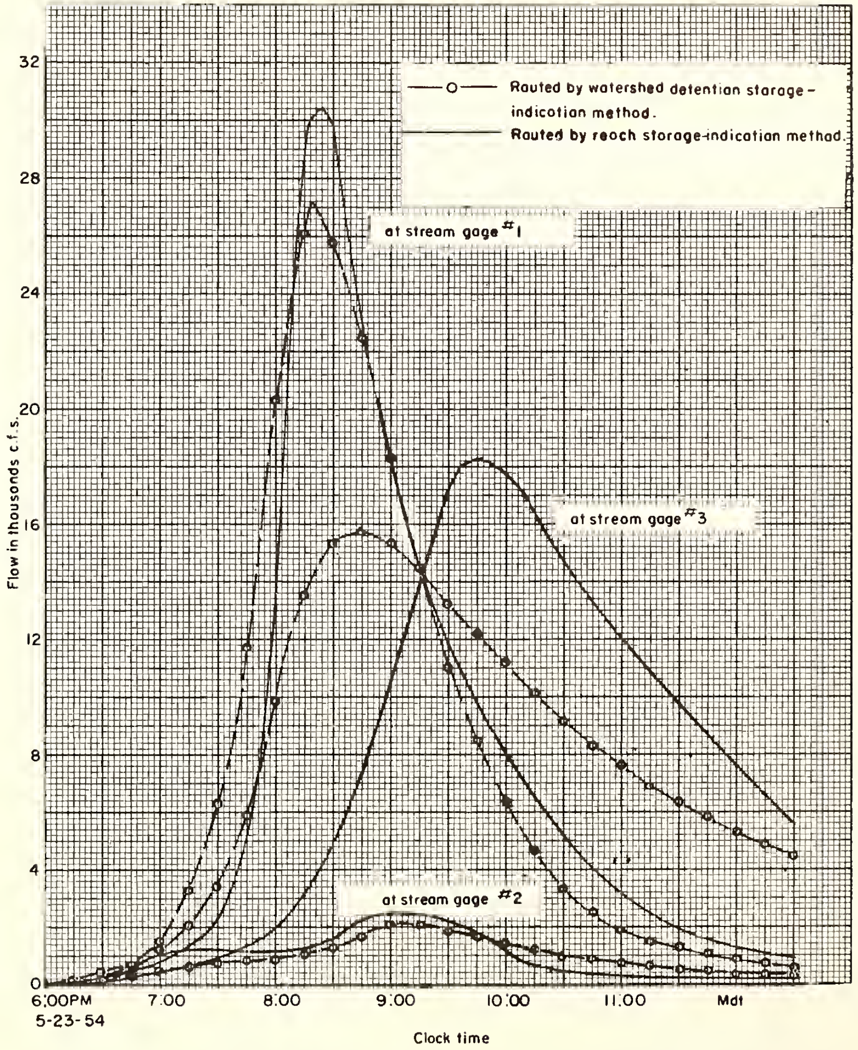

Figure 15 C.--Hydrographs which would have occurred during the storm of May 23 and 24, 1954, if reservoirs had not been installed.

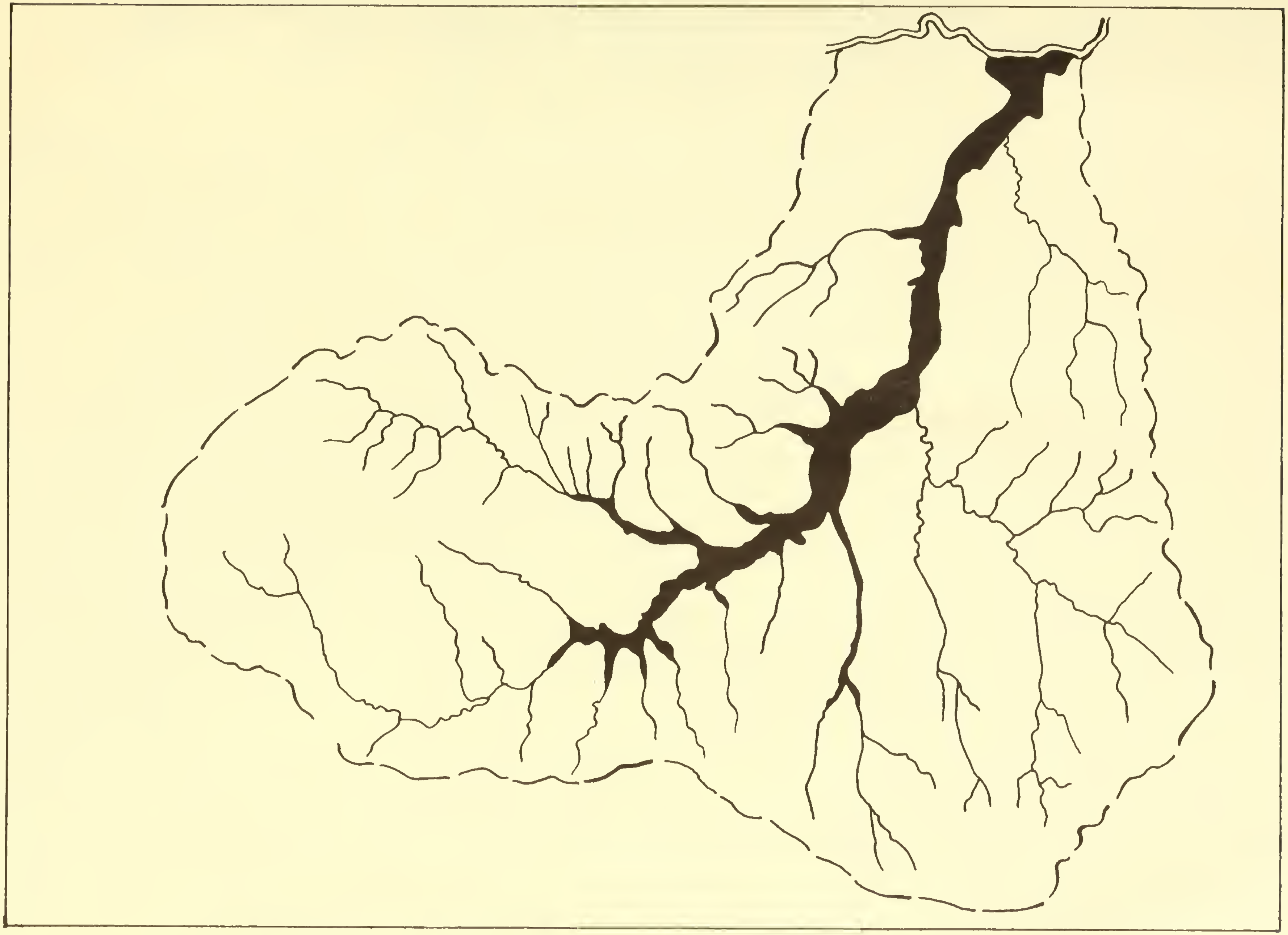

Figure 16.--Area which would have been inundated by the storm of April 29 and 30, 1954, if the reservoirs had not been installed.

for the April storm and 7 percent for the May storm (table 2).

Results from all three methods of computing inundated area are within 6 percent of the average of all three. From these calculations, it would appear that the estimates developed in the work plan are well substantiated.

Effects of the Flood-Reduction Program

By comparison with the averages of estimates by three methods, the actual inundation during the April storm was 25 percent of the area which would have been inundated without protective measures. For the May storm actual inundation was 9 percent of what would have occurred without the program. This is a reduction of 75 percent and 91 percent respectively for these two storms due to the program.

The 588 acres of flood plain inundated in the April storm are approximated in figure 1 by the solid lines representing the stream course. The extent of the estimated flooding by the same storm without the flood-reduction program is shown by the hachured flood-plain area in figure 16. The areas covered by the reservoirs (fig. 1) are not included in the 588 acres of flood plain actually inundated by the April storm. Reservoir sites are selected on relatively low-valued land near the upstream extremities of the flood plain. Quite often the area inundated is more valuable as a stock pond than it was for its previous usage. Also, these areas inundated by detention storage are available still for those uses which can tolerate occasional inundation. The value of this land is included in the cost of the program for evaluating benefit-cost ratios.

The comparison for the May storm is about the same. There were some differences in the specific locations of flood-plain areas flooded but these differences could not be shown on maps of this scale. Figures 1 and 16, therefore, approximately illustrate the effect of the flood-reduction program in the May storm.

The Hammon flood of 1934 inundated the entire 4,700 acres of flood plain, represented by the hachured area on figure 1. The foregoing estimates indicate that without the flood-reduction program the April storm would have inundated 49 percent of the flood plain and the May storm 48 percent. However, with the program in effect the April storm flooded 12.5 percent and the May storm 4.3 percent of the flood plain.

Symbols

The following symbols are used in the equations and graphs throughout this report:

A = Area in acres;

D_a = Volume of detention storage in watershed-surface inches.

E = Excess rainfall, equals $P - (F + V_d + \text{abstractions})$.

F = Amount of infiltration in inches of water.

f = Rate of infiltration in inches per hour.

I_a = Average inflow for an interval Δt, in cubic feet per second.

m = Arithmetic slope of relationship between q and D_a.

O = Outflow, or flow past a given point in cubic feet per second.

P = Amount of precipitation in inches.

Q = Amount of runoff in inches.

q = Rate of runoff in inches per hour.

S = Volume of detention storage in cubic feet per second x hours.

t = Time in hours.

t_c = Time theoretically condensed to make P plot on straight line against time.

V_d = Volume of depression storage in inches.

Δ = Increment.